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NATIONAL DAM SAFETY PROGRAM. SHERANDO DAM (VA 01520), POTOMAC R--ETC(U)
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1 OF 1

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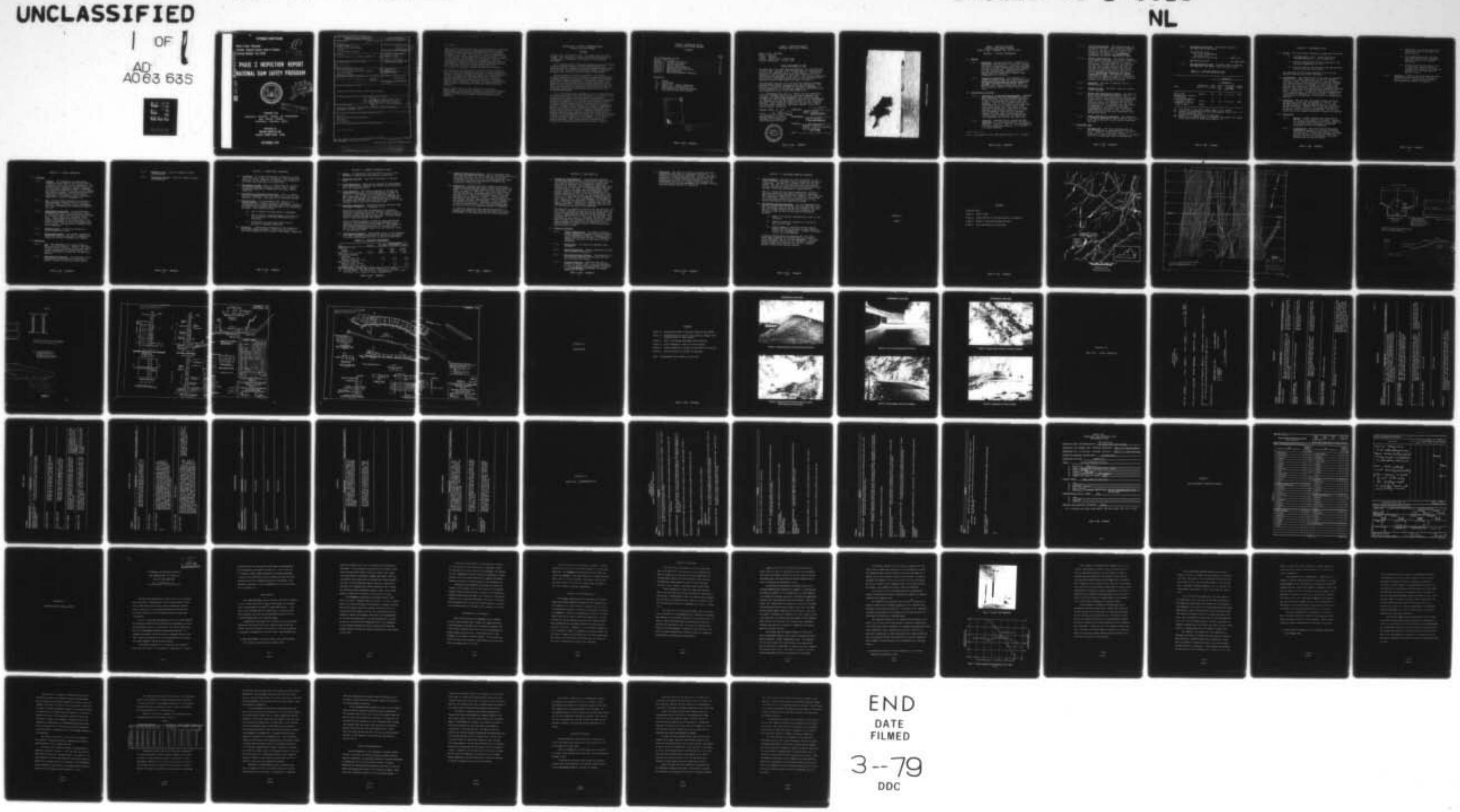

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POTOMAC RIVER BASIN

Name of Dam: Sherando
Location: Augusta County, State of Virginia
Inventory Number: VA 01520

LEVEL

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U.S. ARMY CORPS OF ENGINEERS • NORFOLK DISTRICT •

PREPARED FOR
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

PREPARED BY
MICHAEL BAKER, JR., INC.
BEAVER, PENNSYLVANIA 15009

SEPTEMBER 1978

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: VA 01520	2. GOVT ACCESSION NO.
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4. TITLE (and Subtitle): Phase I Inspection Report National Dam Safety Program Sherando Augusta County, Virginia	5. TYPE OF REPORT & PERIOD COVERED: (9) Final rept.
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Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

(6) National Dam Safety Program. Sherando Dam (VA01520), Potomac River Basin, North Fork of Back Creek, Augusta County, Virginia. Phase I Inspection Report.

18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service, Springfield, Virginia 22151

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dams - VA
National Dam Safety Program Phase I
Dam Safety
Dam Inspection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

(See reverse side)

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

REVISION NO. 1 TO PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

SHERANDO

The cover color is revised to white. The actual cover will not be changed. Each recipient of a copy of this report should notate the existing cover. In addition, add to Section 7, the following paragraphs:

7.1.1 Using the Corps of Engineers screening criteria for initial review of spillway adequacy, it has been determined that the embankment would be overtopped for all storms exceeding approximately 26% of the PMF. The spillway is therefore, adjudged as seriously inadequate and the dam is assessed as unsafe, non-emergency.

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" clsssification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

7.2.1 In accordance with paragraph 7.1.1, it is recommended that within two months from the date of notification to the Governor of the Commonwealth of Virginia, the owner engage the services of a professional consultant to determine by more sophisticated methods and procedures the adequacy of the spillway. Even though the seriously inadequate spillway would produce a dam failure primarily from hydrologic reasons, remedial measures in structural or geotechnical areas may be needed to remove the dam from an unsafe classification. Within 6 months of the date of notification to the governor, the professional consultant's report of appropriate remedial mitigating measures should have been completed and the owner should have an agreement with the Commonwealth of Virginia to a reasonable time frame in which all remedial measures will be complete. In the interim, a detailed emergency operation plan and warning system should be promptly developed. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided.

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

CONTENTS

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Sherando
State: Virginia
County: Augusta
Stream: North Fork of Back Creek
Date of Inspection: 11 July 1978

BRIEF ASSESSMENT OF DAM

Sherando Dam is an earth dam approximately 335 feet long and 38 feet high. The dam, which is operated as a recreational facility, is owned by the U.S. Forest Service. The dam was designed by and constructed under the supervision of U.S. Forest Service personnel in the 1930's. The visual inspection and review of engineering data made in July and August 1978 indicate several items requiring urgent attention.

The emergency spillway will not pass the one-half Probable Maximum Flood without overtopping the dam embankment. The spillway is therefore assessed as seriously inadequate. Visual inspection of the dam was hampered by the heavy brush and vegetation on the lower downstream slope.

It is recommended that the U.S. Forest Service immediately initiate a further study of the spillway inadequacy and reinspect the downstream embankment after removing brush. In addition, the following remedial measures are recommended for completion during the annual maintenance and inspection program: repair spalled concrete in the spillway, operate the lake drain (annually), and check seepage in the drain outlet channel (annually).

MICHAEL BAKER, JR., INC.

Michael Baker, III, P.E.
Chairman of the Board and
Chief Executive Officer

COMMONWEALTH OF VIRGINIA
MICHAEL BAKER III
NO. 3176
CERTIFIED PROFESSIONAL ENGINEER

SUBMITTED: Original signed by JAMES A. WALSH
James A. Walsh
Chief, Design Branch

RECOMMENDED: Original signed by ZANE M. GOODWIN
Zane M. Goodwin
Chief, Engineering

APPROVED: Original signed by: Douglas L. Haller
Douglas L. Haller
Colonel, Corps of Engineers
District Engineer

Date: SEP 25 1978

NAME OF DAM: SHERANDO

OVERALL VIEW OF DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM: SHERANDO DAM, ID# VA 01520

SECTION 1 - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 August 1972 authorized the Secretary of the Army, through the Corps of Engineers to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams. The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Description of Project

1.2.1 Description of Dam and Appurtenances: Sherando Dam consists of an earth embankment approximately 335 feet long and 38 feet high with a 35 feet wide concrete spillway at the left abutment. The 35 feet concrete spillway at elevation 2022 P.D.[1] controls both normal and flood flows using the fixed crest of the concrete entrance weir. The flows are then carried to the natural stream by a 35 feet wide rectangular supercritical concrete channel 350 feet long. Behind the dam is a concrete tower constructed to support the lift mechanism for the 48 inch lake drain.

1.2.2 Location: Sherando Dam is located on Back Creek about 5.3 miles upstream of the Town of Sherando, Virginia. The U.S. Soil Conservation Service's South River Dam No. 27 is located 1.1 miles upstream.

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[1] P.D. - Plan datum is 183$\pm$ feet higher than U.S.G.S. datum.

1.2.3 <u>Size Classification</u>: The maximum height of the dam is 38 feet. The estimated reservoir volume to the top of dam is 393 acre-feet. Therefore, the dam is in the "small" size category as defined by the <u>Recommended Guidelines for Safety Inspection of Dams</u>.

1.2.4 <u>Hazard Classification</u>: The nearest population center to the dam is the Town of Sherando which is located approximately 5.3 miles downstream. Because there is the potential for the loss of many lives in the Town of Sherando and between Sherando and the dam, the dam is considered to be in the "high" hazard category as defined by Section 2.1.2 of the <u>Recommended Guidelines for Safety Inspection of Dams</u>. The hazard classification used to categorize dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 <u>Ownership</u>: The dam is owned by the U.S. Forest Service.

1.2.6 <u>Purpose of Dam</u>: The dam is used for recreational purposes.

1.2.7 <u>Design and Construction History</u>: The existing facility was designed in 1934 by the U.S. Forest Service and constructed in 1935 by the Forest Service using the Civilian Conservation Corps (C.C.C.). An extensive Construction and Soils Report was prepared for the 1935 construction and is included with this report as Appendix VI. According to available correspondence and plans, the spillway was lengthened by 110 feet in 1937. No other known construction has been undertaken since the dam was built.

1.2.8 <u>Normal Operational Procedures</u>: The reservoir is normally operated at the elevation 2022.0 P.D. A 48 inch slide gate can be used to draw down the lake.

1.3 <u>Pertinent Data</u>

1.3.1 <u>Drainage Area</u>: The total drainage area of the Sherando Dam is approximately 4.25 square miles. However, South River Dam No. 27 (located 1.7 miles upstream) controls 2.85 square miles of the drainage area of Sherando Dam.

1.3.2 Discharge at Dam Site: The maximum flood at the dam site is not known.

Principal Spillway:
Pool level at emergency spillway crest 0
Pool level at top of dam . . . . 2844 c.f.s.

Emergency Spillway . . . . Not Applicable

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the following table:

TABLE 1.1 DAM AND RESERVOIR DATA

		Reservoir			
			Capacity		
Item	Elevation feet P.D.(a)	Area acres	Acre-feet (b)	Watershed inches (c) (d)	Length feet
Top of dam	2030.8	32	393	1.7 (5.3)	3010
Maximum pool, design surcharge	-	-	-	-	-
Principal-emergency spillway crest	2022.0	20	170	0.8 (2.3)	2006
Streambed at center-line of dam	1993±	-	-	-	-

(a) Plan datum is 183± feet higher than U.S.G.S. datum.
(b) Storage is estimated from field measurements and U.S.G.S. 7.5 minute quadrangle maps.
(c) Based on 4.25 square miles of watershed.
(d) Based on 1.4 square miles of watershed, excluding 2.85 square miles controlled by South River Dam No. 27.

SECTION 2 - ENGINEERING DATA

2.1 Design: The design data reviewed included the following:

1) Correspondence of U.S. Forest Service on Sherando Dam Project, 1934 through 1937.

2) Design drawings and as-built drawings of the spillway lengthening in 1937.

3) Design Plans for the original dam and spillway revision during construction.

All pertinent existing data has been filed with the Norfolk District for future reference.

2.2 Construction: The construction of the dam was completed by the C.C.C. under supervision of the U.S. Forest Service in 1935. During construction, several changes were made including the decision to narrow the spillway from 55 feet wide to its present 35 feet width. The spillway was originally constructed with flashboards. However, they were later replaced by a permanent concrete weir. On-site soil testing was conducted during the entire construction. A detailed Construction Memorandum is attached as Appendix VI of this report. Photos of construction are available, but were not of reproducible quality.

2.3 Operation: The dam does not require manual operation as the fixed crest weir discharges all inflow. The emergency gate is manually operated by U.S. Forest Service personnel. Maintenance on the dam is supplied by the U.S. Forest Service. No records are kept of reservoir levels or spillway discharges. The 48 inch slide gate is not opened routinely to check its function.

2.4 Evaluation

2.4.1 Design: Design drawings were available at the time of this report for review. However, an in-depth evaluation of structural stability was not available; therefore, a detailed assessment can not be completed.

2.4.2 Construction: The Construction Memorandum attached as Appendix VI provides a detailed description of construction methods, including quality control. The as-built conditions of Sherando Dam differ considerably from the design plans as outlined below.

1) Additional fill was placed on the downstream slope of the dam for a parking area.

2) The flashboards in the spillway have been replaced by a permanent concrete weir at the original elevation.

3) The spillway was originally to be 55 feet wide; however, during construction the spillway width was changed to 35 feet as shown on Plates 3 and 4.

2.4.3 Operation: The 48 inch slide gate should be formally checked periodically, perhaps as part of the U.S. Forest Service's Annual Maintenance and Inspection Program.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

3.1.1 General: The embankment and appurtenant structures were found to be in good overall condition at the time of the inspection. The problems noted during the visual inspection do not require immediate remedial treatment, but should be corrected as part of the regular maintenance program. Noteworthy deficiencies observed are described in the following paragraphs. The complete visual inspection check list is given in Appendix III.

3.1.2 Dam: No noticeable seepage was observed in the embankment; however, there is a heavy growth of trees and brush on the downstream face from the parking lot bench area down to the toe.

3.1.3 Appurtenant Structures: In the principal-emergency spillway, the construction joints in the bottom slab of the spillway are spalled up to three inches wide and three inches deep. Clear seepage was observed in the channel downstream of the outlet pipe. The water was spouting one inch high between the stone joints. The quantity of seepage could not be measured.

3.1.4 Reservoir Area: No serious shoreline or gully erosion was noted.

3.1.5 Downstream Channel: The channel immediately downstream from the spillway is composed of riprap and natural streambed with cobbles.

3.2 Evaluation

3.2.1 Dam: The embankment is in good condition with only minor erosion. However, the heavy growth of trees and brush on the lower downstream slope should be removed during the annual maintenance program and the dam reinspected.

3.2.2 Appurtenant Structures: The spalling of the construction joints should be repaired to prevent further erosion and deterioration.

3.2.3 <u>Reservoir Area</u>: Does not require further investigation.

3.2.4 <u>Downstream Channel</u>: Does not require further investigation.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: No formal operational procedures are used since this dam is used for recreation. The normal pool elevation of the reservoir is controlled by overflow of the three feet high weir in the spillway.

4.2 Maintenance of Dam: The U.S. Forest Service, through the George Washington National Forest, has a yearly maintenance program in conjunction with their annual inspection.

4.3 Maintenance of Operating Facilities: The U.S. Forest Service is responsible for the operation of the 48 inch emergency lift gate. The gate is not routinely opened.

4.4 Warning System: At the present time, there is no warning system or evacuation plan in operation. It is recommended that a formal emergency procedure be prepared, and prominently displayed and furnished to all operating personnel. This should include:

1) How to operate the dam during an emergency.

2) Who to notify, including public officials, in case evacuation from the downstream area is necessary.

3) Procedures for evaluating inflow during periods of emergency operation.

4.5 Evaluation: Although the maintenance of the operating facilities appears adequate, formal records of lift gate checks should be made a part of the annual inspection.

SECTION 5 - HYDRAULIC/HYDROLOGIC DATA

5.1 Design: No design data was available for use in the analyses of hydrologic and hydraulic conditions.

5.2 Hydrologic Records: None were available at the dam site.

5.3 Flood Experience: There are no records of flood experience available for Sherando Dam except during construction and flashboard tests.

5.4 Flood Potential: The flood potential of the dam was evaluated by routing various hydrographs as shown in Table 5.1. The effects of the South River Dam No. 27 were considered in the flood routings for Sherando Dam. The inflow hydrographs for Sherando Dam were obtained by adding the outflow from South River Dam No. 27 to the computed inflow from the intervening area.

5.5 Reservoir Regulation: Pertinent dam and reservoir data are shown in Table 1.1, paragraph 1.3.3.

Regulation of flow from the reservoir is automatic. Flows are controlled by a free overfall crest in the spillway at elevation 2022.0 P.D. All outflow from the reservoir passes through this spillway, with the exception of flow through the lake drain.

Outlet discharge capacity, reservoir area and storage capacity, and hydrograph and routing determinations were calculated as part of this report. The routing of the Probable Maximum Flood (P.M.F.), one-half P.M.F., and 100 year hydrographs began with the reservoir level at the spillway crest.

5.6 Overtopping Potential: The probable rise in the reservoir and other pertinent information on reservoir performance in various hydrographs is shown in the following table:

TABLE 5.1 RESERVOIR PERFORMANCE

Item	Normal	Hydrographs 100 Year	 1/2 P.M.F.	 P.M.F.
Peak flow, c.f.s.				
Inflow	-	750	5330	10,850
Outflow	-	429	5269	10,779
Peak elev., ft. P.D.(a)	2022.0	2024.6	2032.6	2034.7
Principal-emergency spillway				
Depth of flow, ft.	-	1.7	7.0	8.5
Average velocity, f.p.s.	-	7.4	15.0	16.5
Non-overflow section				
Depth of flow, ft.(b)	-	-	1.0	2.3
Average velocity, f.p.s.	-	-	5.7	8.6
Tailwater elev., ft. M.S.L.	-	-	-	-

(a) Plan datum is 183± feet higher than U.S.G.S. datum.
(b) Average depth. The duration of overtopping for P.M.F. and 1/2 P.M.F. is 5.1 and 3.3 hours, respectively.

5.7 Reservoir Emptying Potential: The 36 inch metal pipe entering the reservoir at the tower at elevation 1995.3± P.D. and controlled by a 48 inch slide gate will permit withdrawal of about 161 c.f.s. with the reservoir level at the spillway crest and essentially dewater the reservoir in about one day.

5.8 Evaluation: Sherando Dam with a "small" size-"high" hazard classification must pass a spillway design flood equal to a value between one-half P.M.F. and P.M.F. As shown in Table 5.1, the P.M.F. was routed and found to overtop the dam by an average depth of 2.3 feet. The one-half P.M.F. was also routed and found to overtop the dam by an average of 1.0 feet. A flood equal in magnitude to the 100 year flood was also routed and did not overtop the dam. The spillway, in fact, passes 26 percent of the P.M.F. Therefore, since the P.M.F. as well as the one-half P.M.F. overtop Sherando Dam, the spillway must be considered seriously inadequate.

It should be indicated that conclusions pertain to present day conditions, and that the effect of future development on the hydrology has not been considered.

SECTION 6 - DAM STABILITY

6.1 Foundation and Abutments: Cross sections showing borings and test pits, and a preliminary report of the soil investigations with construction control were available to determine the foundation conditions. Based on the borings, the preliminary report and field observations at the site, it was determined that the alluvial soils are sandy loam including sand, clay, gravel, cobbles, boulders and rock fragments with depths averaging 10 feet in the valley lowland. The residual and colluvial soils on the slopes consist of clay, silt, sand and rock fragments at depths usually more than 10 feet. The right abutment is founded on soil, sandstone and quartzite; whereas, the left abutment of the dam is located at a concrete wall for the ungated spillway. The borings indicate that the cutoff trench was excavated into sandstone.

The bedrock, as exposed in the cut for the spillway and a few small outcrops in the hill on the right side, is very hard quartzite and hand quartzose sandstone. The strike is approximately parallel to the spillway and a variable dip of 5° to 10° east into the spillway. The thickness of the strata is variable, and the two major sets of joints have created a blocky structure. Minor clear seepage was observed at the exposure of the bedding planes in several areas of the spillway cutslope in the hillside.

6.2 Stability Analysis

6.2.1 Visual Observations: No tension cracks or other evidence of movement such as sloughing of the embankment slopes or movement at or beyond the toe were noticed. No seepage was observed on the face of the dam or at the abutments.

6.2.2 Design Data: No stability analyses were available.

6.2.3 Operating Records: Recent inspection records are included in Appendix V.

6.2.4 Post-Construction Changes: The extension of the spillway appears to have been the only post-construction change.

6.2.5 Seismic Stability: Sherando Lake Dam is located in Seismic Zone 2 and is considered to have no hazard from earthquakes according to the Recommended Guidelines for Safety Inspection of Dams.

6.3 <u>Evaluation</u>: The lack of stability analyses and soil properties make stability assessment difficult. No sloughing or seepage was observed. However, the thick vegetation on the downstream slope and abutment made observation of these conditions difficult. It is recommended that the U.S. Forest Service remove the trees and bush from the embankment and abutment contact, and make another surficial inspection.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: The dam will pass 26 percent of the P.M.F. without overtopping and is assessed as seriously inadequate. The dam will, however, pass the 100 year flood. The lack of available stability calculations for the dam and the heavy brush on the downstream slope prevent a thorough assessment of dam stability.

The U.S. Forest Service should immediately initiate a detailed hydrologic and hydraulic study to determine the increase in required spillway capacity for compliance with applicable regulations. In addition, other items as listed below should be completed as part of the annual maintenance and inspection program.

7.2 Recommended Remedial Measures: It is recommended that the U.S. Forest Service remove the trees and brush from the downstream slope of the dam and reinspect the area. The U.S. Forest Service should also accomplish the following items as part of the annual maintenance program:

1) Repair the spalled construction joints in the spillway.

2) Annually check the operation of the slide gate on the tower.

3) Check seepage in the drain outlet channel monthly and during periods of high reservoir levels for increased flow. Further investigation is also recommended.

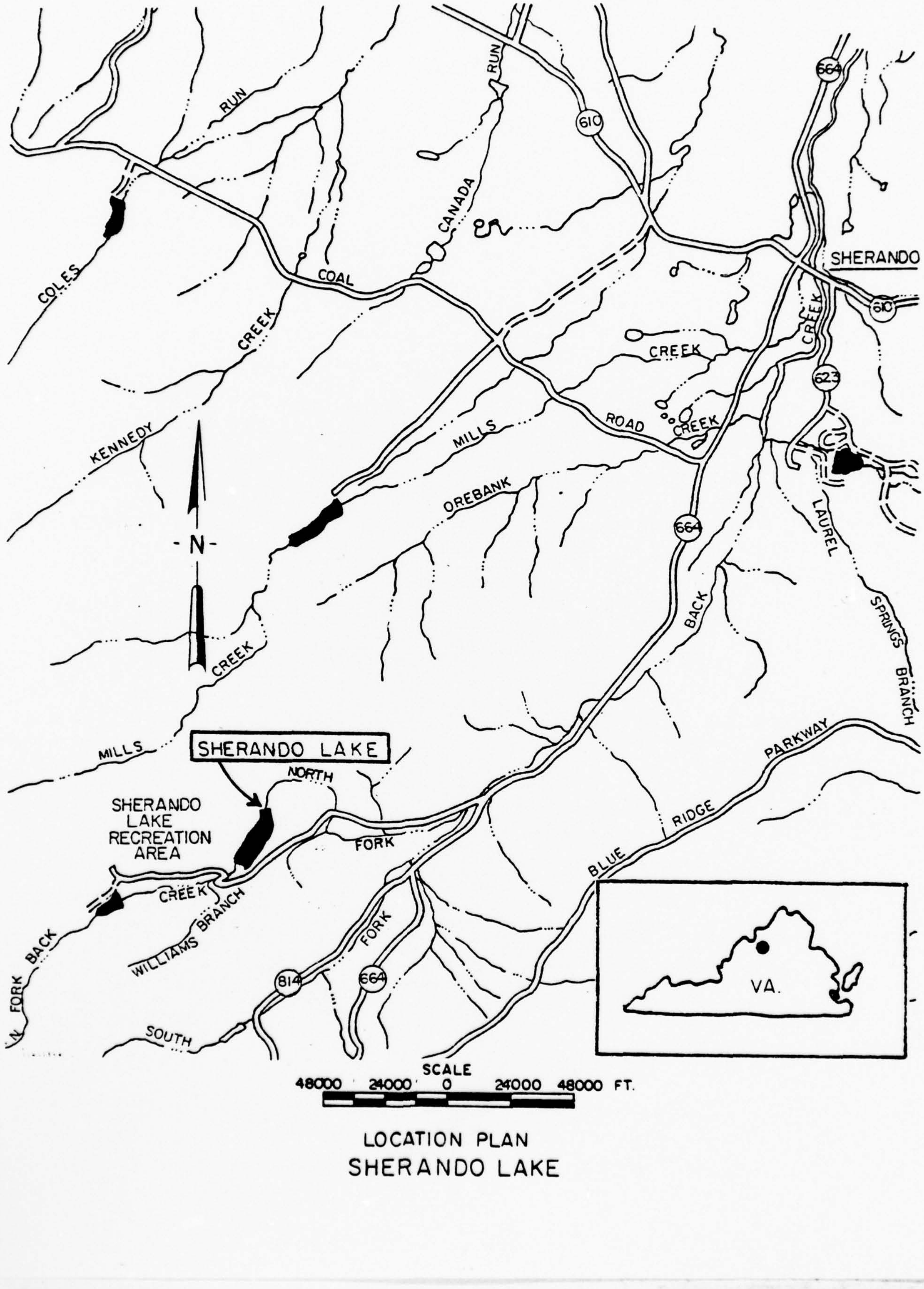
A warning system should be devised that will alert downstream occupants to evacuate when the reservoir level approaches the top of the embankment. The downstream occupants should also be advised to evacuate during storms that coincide with the U.S. Weather Bureau's flash flood warning system.

APPENDIX I

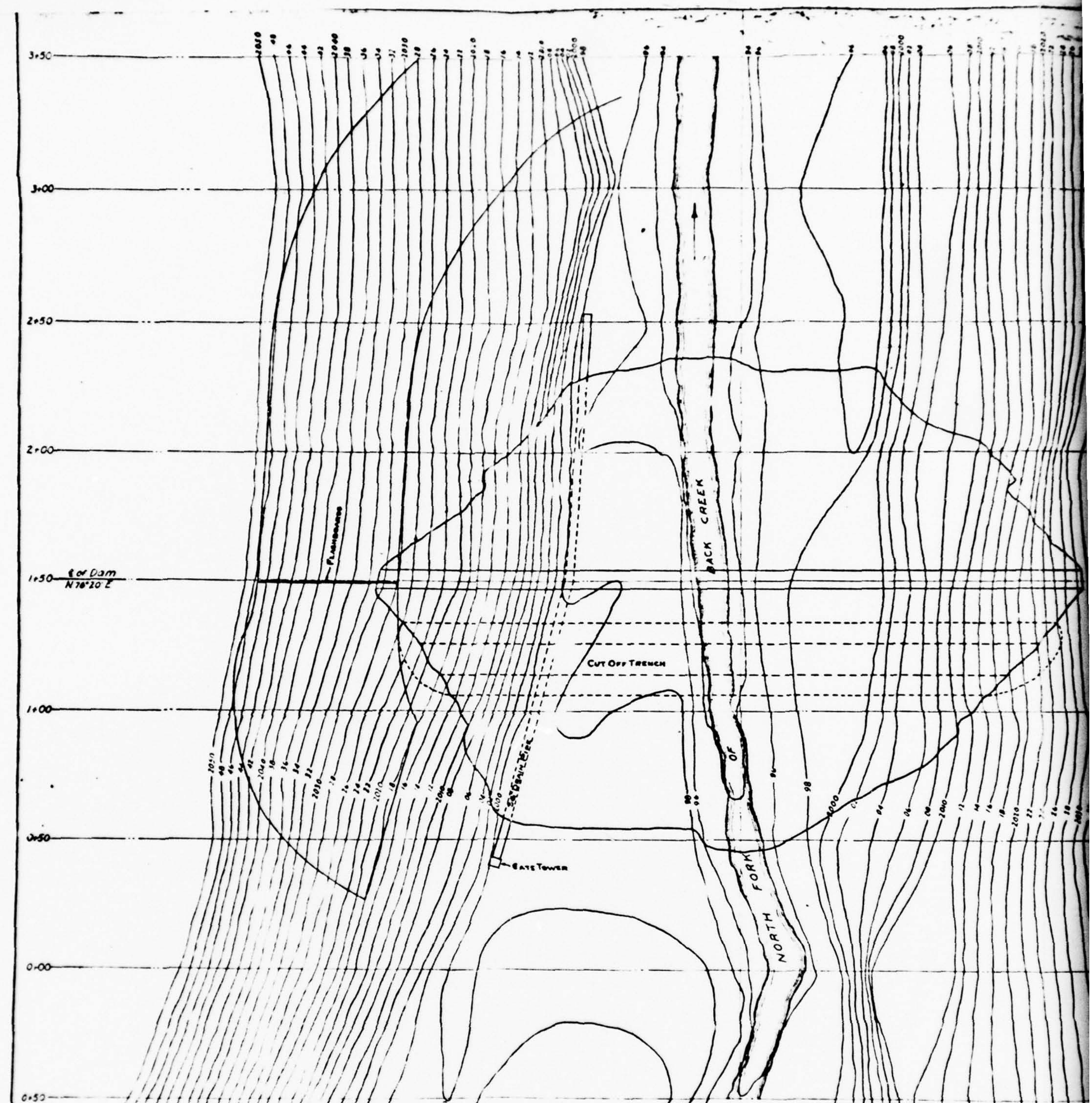
PLATES

CONTENTS

NAME OF DAM: SHERANDO



LOCATION PLAN
SHERANDO LAKE


3+50
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2+50
2+00
1+50
1+00
0+50
0+00
0-50
℄ of Dam
N 76°20' E
FLASHBOARDS
CUT OFF TRENCH
GATE TOWER
BACK CREEK
NORTH FORK OF

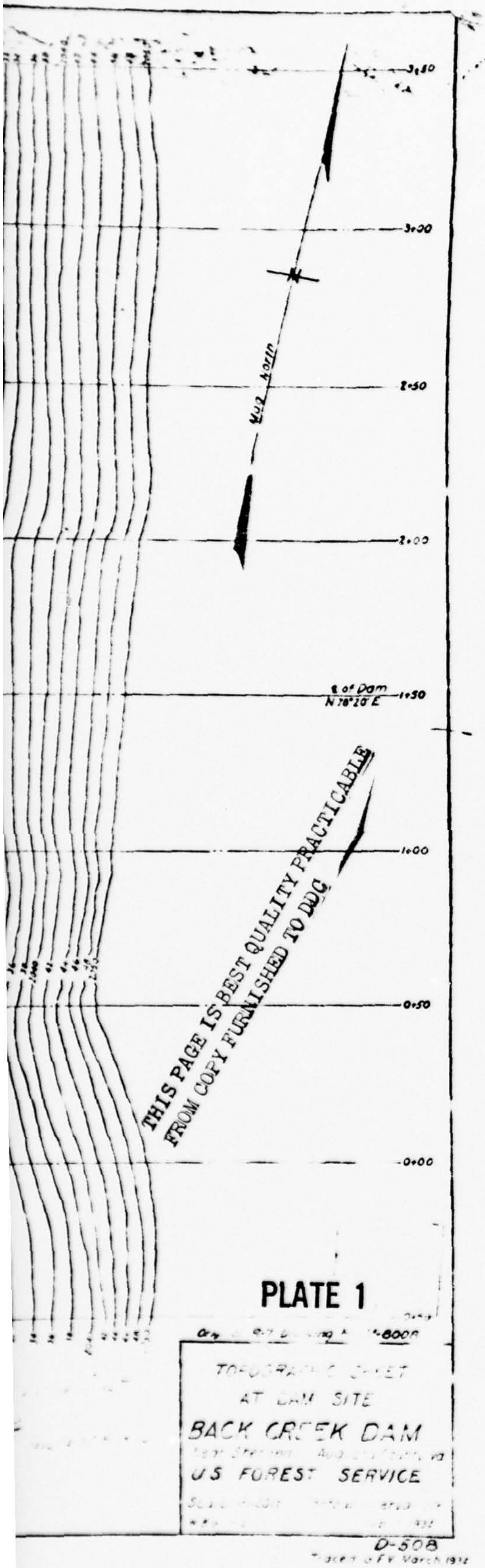
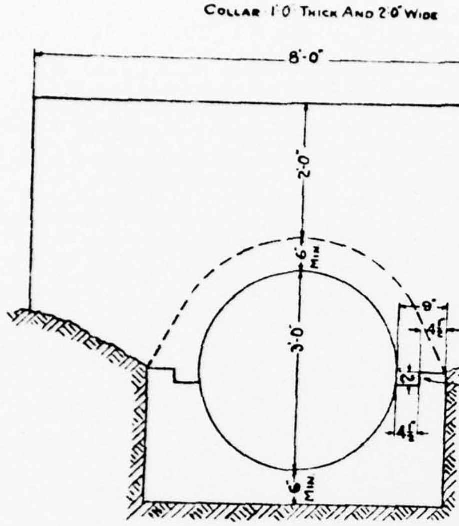
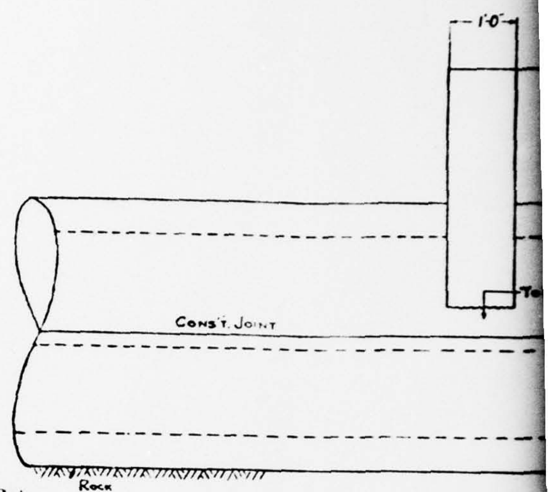

3+50
3+00
2+50
2+00
℄ of Dam
N 78°20' E
1+50
1+00
0+50
0+00
TOPOGRAPHIC SHEET
AT DAM SITE
BACK CREEK DAM
US FOREST SERVICE
D-508

PLATE 1

2

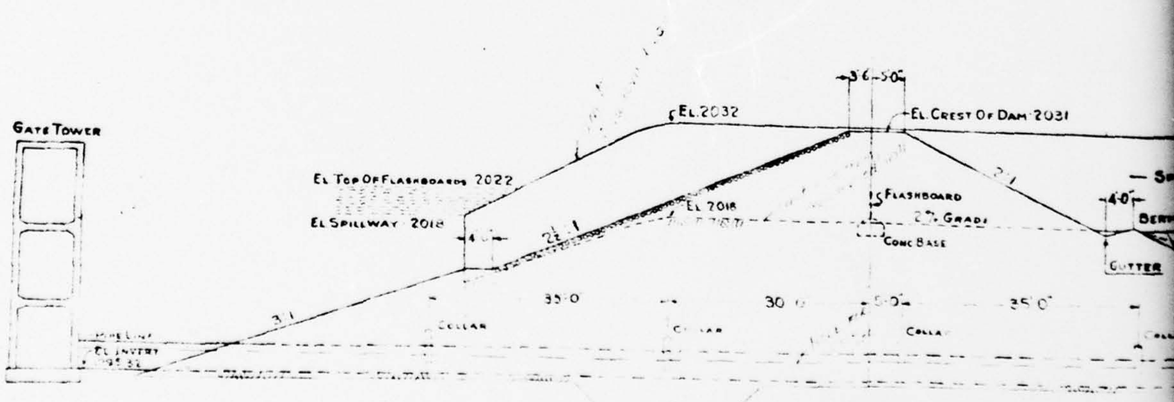


Cross Section Of Drain Pipe
At Collar
Scale - 1" = 1'-0"



— Section On Pipe Line Showing Collar —

NOTE: Pipe Line To Rest On Solid Rock. Four Collars, One Foot Thick By Two Feet Wide Are To Be Poured As A Part Of The Pipe Line. The First Collar Is Located 65' Upstream From Axis Of Dam. The Other Three Are Spaced At Intervals Of 35'



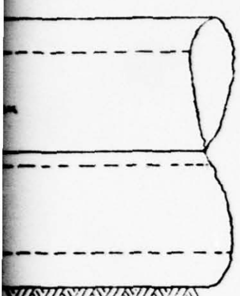
— Cross Section Of Dam —

Scale 1" = 10'-0"

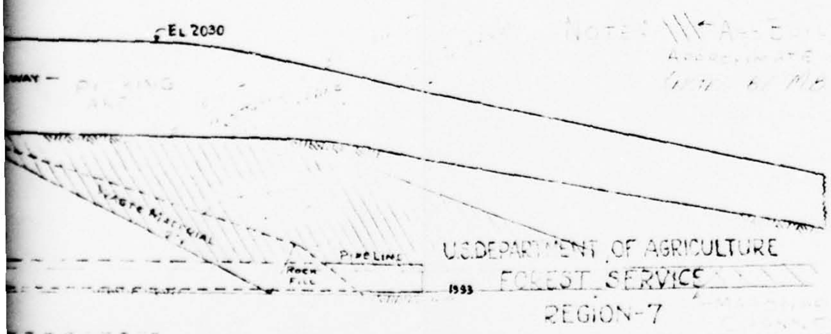
Dwg. No.

Est. Vol. of Material in Back Creek Dam

Elevation		Cu. Yards	
1996		378	
1998		1240	
2000		2740	
2002		4470	
2004		6200	
2006		7920	
2008		9810	
2010		11200	
2012		12800	
2014		14300	
2016		15700	
2018		16900	
2020		18100	
2022		19200	
2024		20000	
2026		20700	
2028		21200	
2030		21400	
2032		21800	
2'-0" into Hillside	= 2710	24310	
Core of Dam	= 1140	25450	Total



NOTE: Pipe Line, Gate-Tower And Spillway
Wall To Be Class 'A' Concrete.
Materials, Mixing, Placing And Curing In
Accordance With Detailed Concrete
Specifications In Region 7 Specifications
For Minor Forest Road Construction.
Water Ratio - 6 Gals. Water Per Bag Of Cement
Approximate Mixture By Volume - 1:2:3½

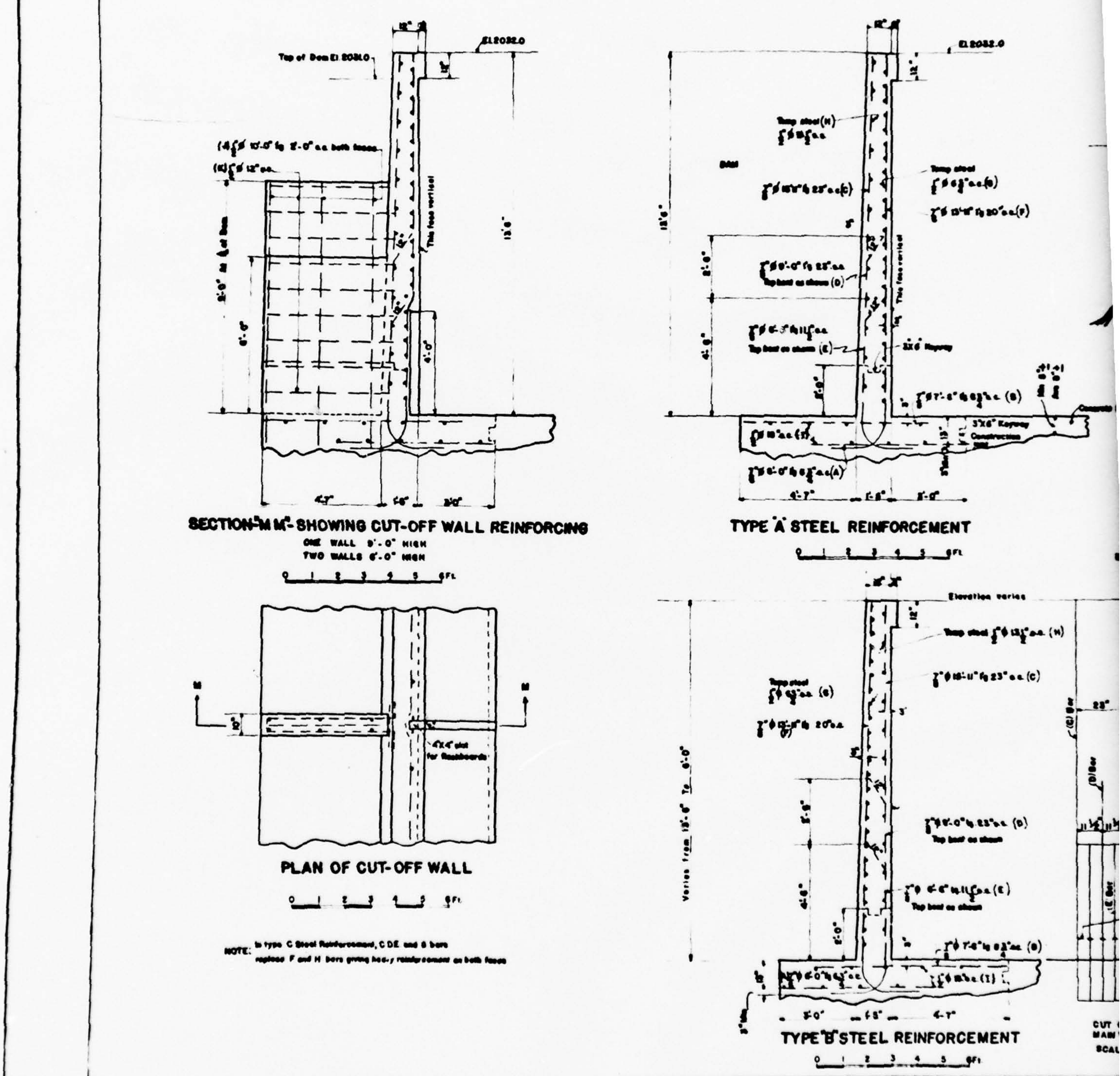


U.S. DEPARTMENT OF AGRICULTURE
FOREST SERVICE
REGION-7
CROSS SECTION OF DAM AND DETAIL OF PIPE LINE
BACK CREEK DAM - AUGUSTA CO. VA.
GEO. WASHINGTON NATIONAL FOREST

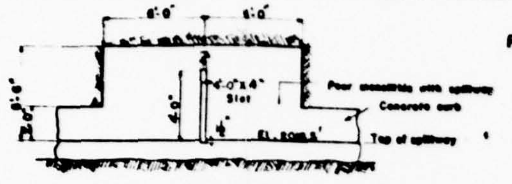
DESIGNED ______ CHECKED ______
DRAWN ______ TRACED ______
APPROVED ______ DATE JUNE 1934
Reg. Eng.

PLATE 2

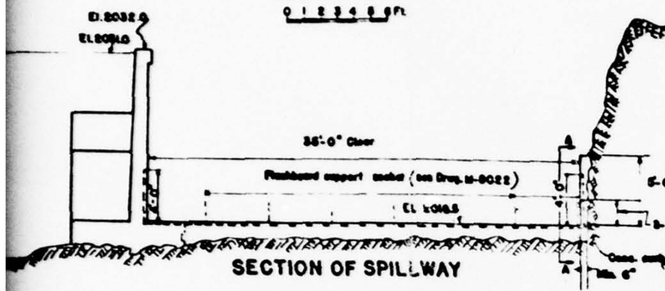
2


SECTION "M-M" SHOWING CUT-OFF WALL REINFORCING
ONE WALL 9'-0" HIGH
TWO WALLS 6'-0" HIGH
TYPE "A" STEEL REINFORCEMENT
PLAN OF CUT-OFF WALL
TYPE "B" STEEL REINFORCEMENT
NOTE: In type C Steel Reinforcement, C D E and G bars replace F and H bars giving heavy reinforcement on both faces
This face vertical
Top bent as shown
3"X6" Keyway
Construction Joint
Elevation varies
Varies from 13'-6" To 6'-0"

R-7-DRAWING № D 521
REVISION OF DRAWING- D 516



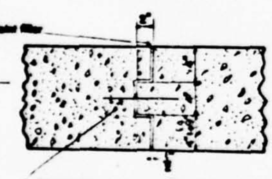
SECTION A-A



SECTION OF SPILLWAY

REINFORCING STEEL SPECIFICATIONS

F.S.B.-QQ-B-71, Grade-1
Type-B-(Deformed Structural) round
Class-A-(Open Hearth Steel)



DETAIL-EXPANSION JOINT

REINFORCING SCHEDULE

SYMBOL	SIZE	DESCRIPTON	NO BARS	LENGTH OF EACH BAR	TOTAL LENGTH 7/8" φ	TOTAL LENGTH 1/2" φ	TOTAL WEIGHT LBS
A	7/8" φ	Straight	510	6'-0"	3060	—	
B	7/8" φ	Straight	395	7'-6"	2963	—	
C	7/8" φ	Varies 15'-7" to 6'-1"	172	Varies 16'-4" to 8'-10"	2889	—	
D	7/8" φ	6'-3"	170	10'-0"	1700	—	
E	7/8" φ	3'-9"	340	7'-6"	2550	—	
F	7/8" φ	Varies 15'-7" to 6'-1"	150	Varies 16'-4" to 8'-10"	2165	—	
G	1/2" φ	Straight				6936	
H	1/2" φ	Straight				2498	
I	1/2" φ	Straight				2940	
J	1/2" φ	Straight	6 12	10'-0" 7'-0"		144	
K	1/2" φ	6" 4'-10" 6" 4'-10" 8"	21	11'-2"		235	
		For possible extension			1503	2781	
	7/8" φ	TOTALS			16800		34000
	1/2" φ	TOTALS				15500	10400

NOTE: -1- No reinforcing to extend thru construction joints.
2- For PLAN and ELEVATION of retaining wall see dwg. M-8022

PLATE 3

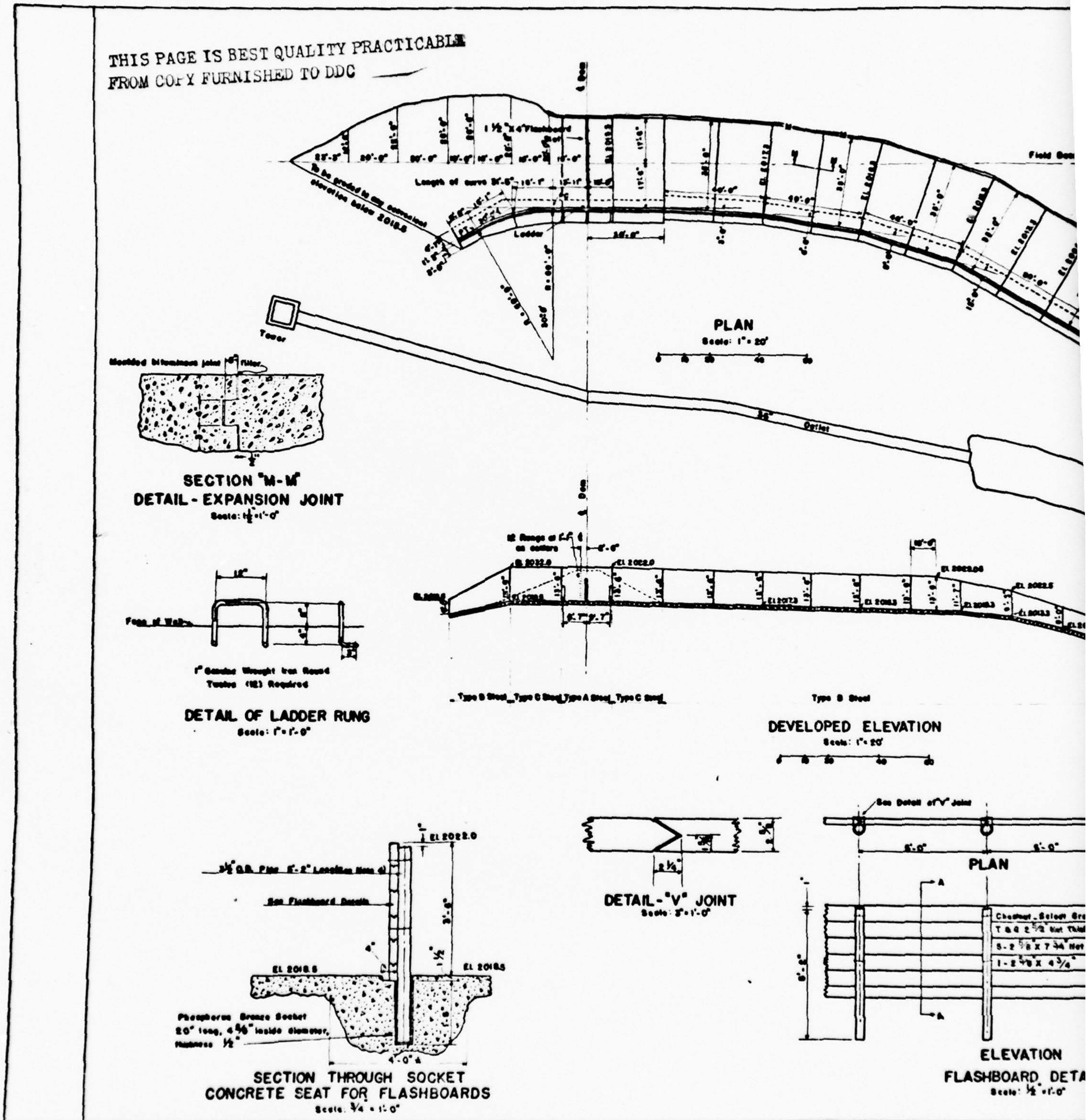
U. S. DEPARTMENT OF AGRICULTURE
FOREST SERVICE
REGION 7
DETAIL OF SPILLWAY RETAINING WALL
BACK CREEK DAM
GEORGE WASHINGTON NATIONAL FOREST

DESIGNED A.H.R. DRAWN A.H.R.
CHECKED H.C.B. TRACED J.C.M.
RECOMMENDED FOR APPROVAL C.H. Betts DATE March 16, 1935
APPROVED [signature] REGIONAL ENGINEER

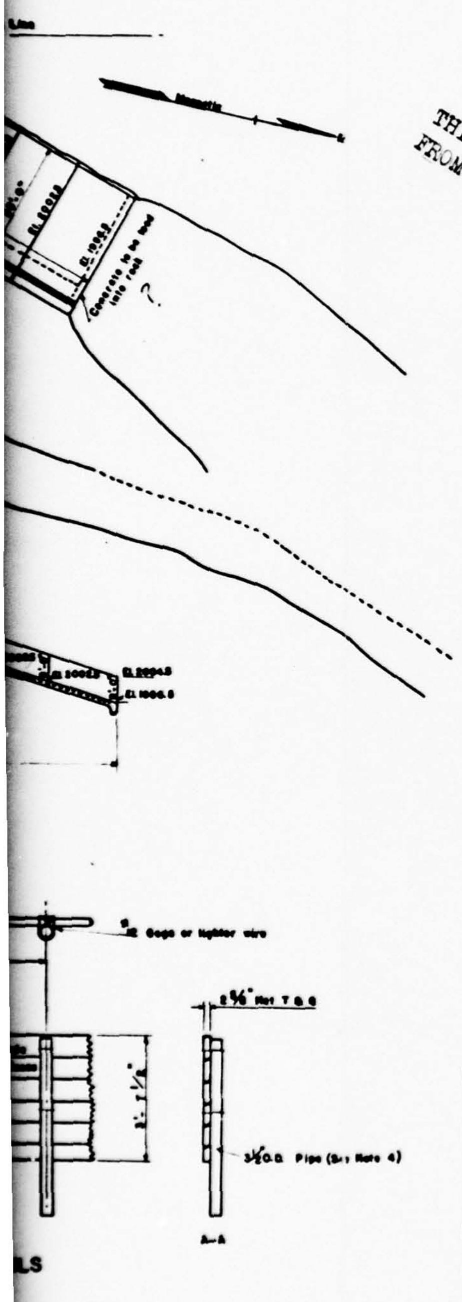
DATE
DRAWING No.

D 521

2


PLAN
Scale: 1" = 20'
Tower
Ladder
Length of curve
Outlet
SECTION "M-M"
DETAIL - EXPANSION JOINT
DETAIL OF LADDER RUNG
Scale: 1" = 1'-0"
Face of Wall
Twelve (12) Required
Type B Steel
DEVELOPED ELEVATION
Scale: 1" = 20'
DETAIL - "V" JOINT
Scale: 3" = 1'-0"
See Detail of "V" Joint
PLAN
ELEVATION
FLASHBOARD DETA
Scale: 1/2" = 1'-0"
See Flashboard Details
EL 2022.0
EL 2018.5
Phosphorus Bronze Socket
SECTION THROUGH SOCKET
CONCRETE SEAT FOR FLASHBOARDS
Scale: 3/4 = 1'-0"

R-7-DRAWING Nº D522



NOTES

1. For details of retaining wall see Dwg. M-8021.
2. Floor of spillway to be paved with plain Class B concrete having a minimum average thickness of 6".
3. Retaining wall to be reinforced Class A concrete.
   Class A: Mix by volume = 1:2:3¼
   6 gals. of water per sack of cement
   Slump 3"
   Class B: Mix by volume = 1:2½:4¼
   7 gals. of water per sack of cement
   Slump 2" or less
4. All pipe to be standard galvanized wrought iron.

DATE

DRAWING

No.

PLATE 4

U.-S. DEPARTMENT OF AGRICULTURE
FOREST SERVICE
REGION 7

SPILLWAY AND RETAINING WALL
BACK CREEK DAM
GEORGE WASHINGTON NATIONAL FOREST

DESIGNED AHR | DRAWN AHR
CHECKED AHR | TRACED N E S
RECOMMENDED FOR APPROVAL | DATE April 17 1935
[signature] | APPROVED [signature]

D 522

2

APPENDIX II

PHOTOGRAPHS

CONTENTS

Note: Photographs were taken 11 July 1978.

NAME OF DAM: SHERANDO

SHERANDO LAKE DAM

PHOTO 1. Downstream Slope of Dam With Parking Area Bench

PHOTO 2. Downstream Drain Outlet Into Masonry Channel
With Artesian Flow in Small Holes

SHERANDO LAKE DAM

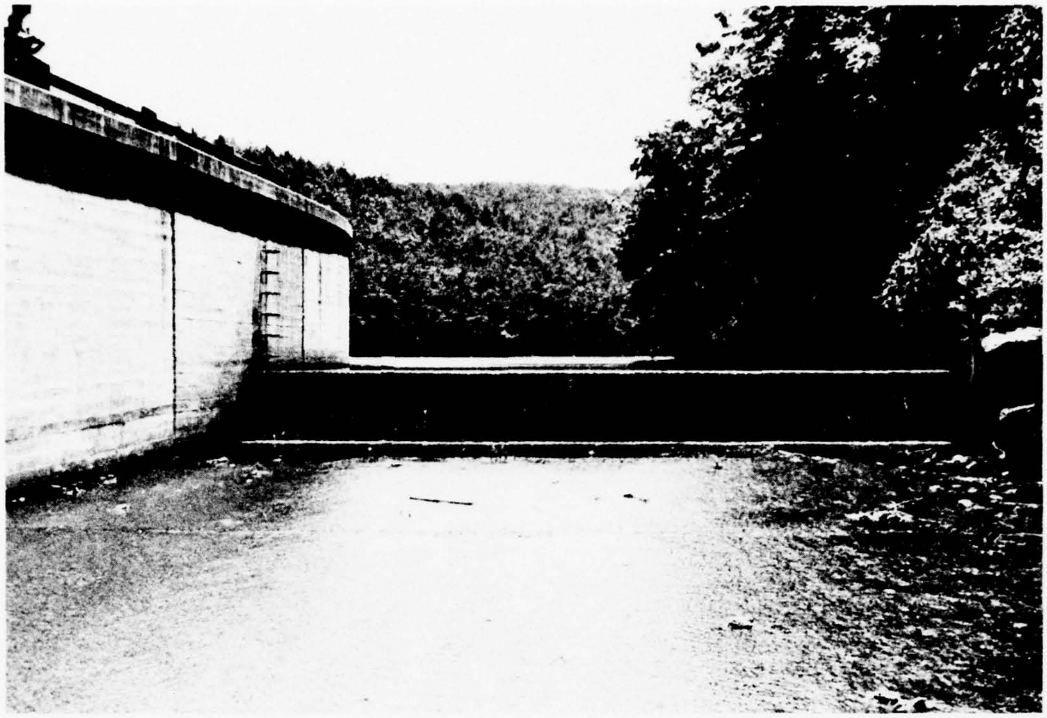


PHOTO 3. Weir in Principal Spillway With Overflow

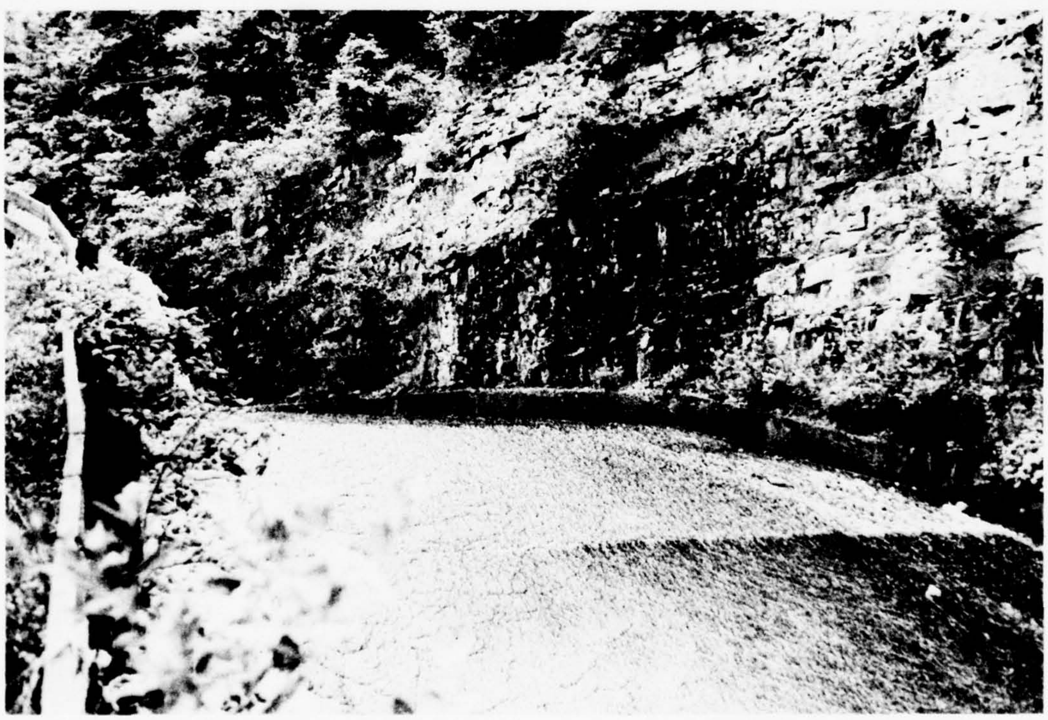


PHOTO 4. Clear Seepage in Rock Cut for Spillway

SHERANDO LAKE DAM

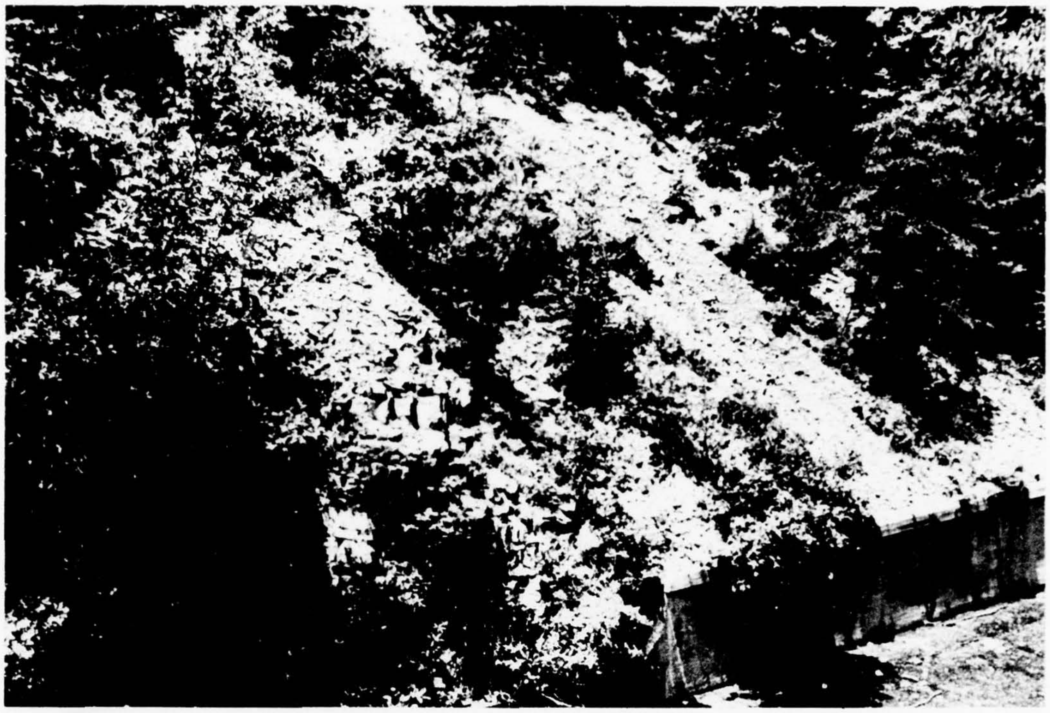


PHOTO 5. Surface Erosion of Slope on Left Bank of Spillway

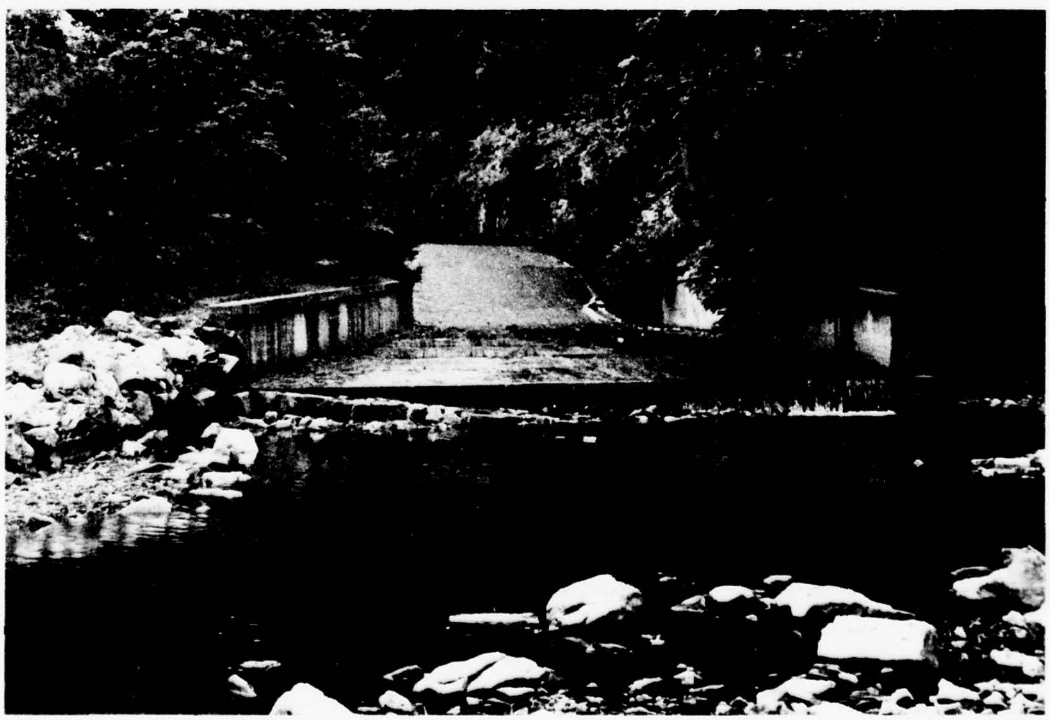


PHOTO 6. Stilling Basin at Outlet of Spillway

APPENDIX III

CHECK LIST - VISUAL INSPECTION

Check List
Visual Inspection
Phase 1

Name Dam Sherando   County Augusta   State Virginia   Coordinates Lat. 3755.5 Long. 7900.2

Date Inspection 11 July 1978   Weather Rain/Cloudy   Temperature 80°F.

Pool Elevation at Time of Inspection 2022.0 P.D.*   Tailwater at Time of Inspection 1993± P.D.*

*P.D. indicates plan datum, approximately 183± feet higher than U.S.G.S.

Inspection Personnel:

MICHAEL BAKER, JR., INC.:	U.S. FOREST SERVICE:
D. J. Greenwood	Richard Graves
J. M. Thompson	
W. L. Sheafer	

D. J. Greenwood Recorder

EMBANKMENT

SHERANDO

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	There were no cracks seen during the inspection. The detection of any cracking below the parking lot level was next to impossible due to the heavy growth of brush and trees.	The dam should be reinspected after the brush and trees are cleared from the lower portions.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	There was no movement or cracking observed.	The dam should be reinspected after the brush and trees are cleared from the lower portions.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	There was no sloughing; however, there was minor erosion around the downstream steps and some removal of topsoil over stone riprap on the upstream slope.	The dam should be reinspected after the brush and trees are cleared from the lower portions.
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	The alignments are good.	
RIPRAP FAILURES	There were no failures in the flat stone riprap on the upstream slope.	Apparently the riprap has been covered by topsoil and seeded for appearance purposes. Several areas have been eroded down to the riprap. This does not cause a structural problem; however, it could be reseeded if desired for cosmetic reasons.

EMBANKMENT

SHERANDO

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONSTRUCTION MATERIAL	Brown, damp, firm, silty sand, gravel and rock fragments were observed at the surface which appeared to be fairly uniform granular material. Information obtained from the U.S. Forest Service indicated that the dam was constructed of these materials.	
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	There appears to be a firm bond at the junctions with the dam. There is a concrete wall for the spillway on the left side and sandy silt with rock fragments overlying quartzose sandstone and quartzite on the right side.	
ANY NOTICEABLE SEEPAGE	No noticeable seepage was observed in the dam. There was heavy growth of vegetation in the lower downstream slope at the toe which made inspection of all areas impossible. Clear seepage was seen in the outlet channel.	The heavy growth near the downstream toe should be cut so that any seepage can more easily be observed.
STAFF GAGE AND RECORDER	There are none.	
DRAINS	There are none.	
FOUNDATION	Apparently the dam was constructed on quartzite and sandy loam with gravel deposits. The dip of the bedrock in the spillway is 5° to 10° east parallel to the centerline of dam and the strike is perpendicular to the dam.	

OUTLET WORKS

SHERANDO

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	The outlet for lake drain is a stone masonry channel in good condition with no major cracking or spalling.	
INTAKE STRUCTURE	The flow out of the dam is controlled entirely by the overflow weir in the spillway. The tower upstream of the dam face is for lake drain operation only.	
OUTLET STRUCTURE	There were traces of minor seepage with a reddish-brown color due to iron precipitate at base of the stone masonry head wall at the end of outlet pipe.	
OUTLET CHANNEL	The channel just downstream from the outlet pipe shows evidence of water under pressure (a clear discharge, coming between the stone joints). The water spouts vertically about one inch at several locations.	The water boiling up should be checked periodically for color change or pressure increase. Further investigation is also recommended.
EMERGENCY GATE	The emergency lake drain is controlled by the inlet tower hand crank. The exit pipe is a 36 inch corrugated metal pipe. From all indications, however, the pipe is controlled by a 48 inch circular sluice gate.	Plans should be made to operate this gate and to evaluate its condition.

UNGATED SPILLWAY

SHERANDO

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	The concrete weir in the spillway is in good condition with only minor erosion from the overflowing water. The weir has a 3.5 feet crest height and is 1.5 feet thick.	
APPROACH CHANNEL		
DISCHARGE CHANNEL	The discharge channel consists of a concrete slab on the bottom, concrete retaining walls on the right side; and a partial concrete retaining wall with exposed rock for the left side. All the retaining walls and slabs are in good condition, except for spalling at the construction joints of the bottom concrete slab. A six inch clay pipe next to the concrete wall and the hillside on left side was dry. The discharge channel has steep gradients and curved sections for transition at the outlet.	
BRIDGE AND PIERS	There are none.	
LEFT CUT SLOPE	The slope is composed of approximately 30 feet of almost vertically cut very hard quartzite and quartzose sandstone which strikes approximately parallel to spillway with variable 5°-10° dip east into spillway. Seepage and runoff of less than one g.p.m. were observed in several areas. Vertical sets of joints are parallel to spillway. Silty sand and rock fragments (large and small) with some sloughing and traces of clear seeps were exposed on downstream slope. There was scattered growth on the slopes. The rock slopes appear stable with minor amounts of small talus.	The rock slopes appear stable; but intermittent sloughing into the channel may occur on the downstream soil slopes in wet seasons.

INSTRUMENTATION

SHERANDO

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	There are none.	
OBSERVATION WELLS	There are none.	
WEIRS	There are none.	
PIEZOMETERS	There are none.	
OTHER		

RESERVOIR

SHERANDO

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	The slopes of silt, sand, gravel and rock fragments are steep and wooded. They appear to be stable. Vertical very hard quartzite and quartzose sandstone strata are exposed in the vicinity of the approach to the ungated spillway. Development is present in the upstream campgrounds.	
SEDIMENTATION	The soundings taken in the reservoir indicate a depth of 17 feet at various locations behind the dam. This depth corresponds to an elevation of about 1822. No estimate is available at the original bottom elevation.	

DOWNSTREAM CHANNEL

SHERANDO

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	The downstream channel is relatively shallow but wide. The channel is composed of cobbles, sand and gravel. There are several bridges crossing the stream between the dam and the Town of Sherando. The channel is in stable condition and generally lacks debris buildup. Generally, the banks of the channel are overgrown by trees.	
SLOPES	The slope of the downstream channel ranges from 85 to 45 feet per mile. The overbanks are flat and vary in width from 100 to 1000 feet. The slopes outside the overbanks rise rapidly to the mountain tops.	
APPROXIMATE NO. OF HOMES AND POPULATION	The nearest inhabitable structures are about 1.8 miles downstream. The structures are sparce along the stream for the next three miles until the Town of Sherando. The estimated population along the reach is about 50. Many of the homes along the stream are at streambank level.	

APPENDIX IV

CHECK LIST - ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

SHERANDO

ITEM	REMARKS
PLAN OF DAM	A complete set of design plans are available at the Norfolk District, Corps of Engineers. A plan view of the dam is included in this report as Plate 1.
REGIONAL VICINITY MAP	The U.S.G.S. 7.5 Minute Sherando, Virginia Topographic Qudrangle was used to prepare the Location Plan.
CONSTRUCTION HISTORY	The dam was constructed by the Civilian Conservation Corps in 1935 under the supervision of the U.S. Forest Service. An extensive Construction Memorandum is included in Appendix VI of this report.
TYPICAL SECTIONS OF DAM	Typical section of the dam is enclosed in the Phase I Inspection Report as Plate 2.
HYDROLOGIC/HYDRAULIC DATA	There were no hydrologic or hydraulic data available for this dam.
OUTLETS - PLAN and DETAILS	are available at the Norfolk District, Corps of Engineers.
- CONSTRAINTS and DISCHARGE RATINGS	are included in the hydraulic and hydrologic calculations for this Phase I report and are on file at the Norfolk District, Corps of Engineers.
RAINFALL/RESERVOIR RECORDS	No rainfall or reservoir level records are available at the dam. Rainfall data is available from Virginia Climatological Records.

SHERANDO

ITEM	REMARKS
DESIGN REPORTS	There were no design reports available for this project.
GEOLOGY REPORTS	A subsurface investigation consisting of test borings are part of the design plans. No formal geology reports were available.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	No design computations were available for this report.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	A Construction Memorandun in Appendix VI and the design plans are the only investigations and/or test records available.
POST-CONSTRUCTION SURVEYS OF DAM	The dam is annually inspected by the U.S. Forest Service.
BORROW SOURCES	Borrow areas are not outlined in available reports on this dam.

ITEM	REMARKS
MONITORING SYSTEMS	No monitoring systems are presently used on the dam, although the design plans show a water level recorder used during flashboard tests.
MODIFICATIONS	The dam received several modifications during construction including: enlargement of the downstream embankment, lengthening the discharge channel, and replacement of the flashboards with a concrete weir.
HIGH POOL RECORDS	No high water records are available.
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Annual inspections are completed by the U.S. Forest Service. No known major construction reports are available for the above changes.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	The dam apparently was damaged by flooding during construction. However, no failures have occurred since completion of construction.
MAINTENANCE OPERATION RECORDS	Annual inspections are made by the U.S. Forest Service. Erosion repair and reseeding has been done.

ITEM		REMARKS
SPILLWAY PLAN		Sections and details of the ungated, concrete spillway are enclosed as Plates 3 and 4.
	SECTIONS	The sections and slopes in the design plans closely match the conditions observed during the visual inspection.
	DETAILS	
OPERATING EQUIPMENT PLANS & DETAILS		Plans and details for the operating equipment are shown on the design plans.

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 2.85 controlled; 4.25 square miles total, wooded

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 2022 P.D.* (170± acre-feet)

ELEVATION TOP OF DAM POOL (STORAGE CAPACITY): 2039.8 P.D.* (393± acre-feet)

ELEVATION MAXIMUM DESIGN POOL: Not Applicable

ELEVATION TOP DAM: 2030.8 P.D.*

CREST: Principal-Emergency Spillway

a. Elevation 2022.0 P.D.*
b. Type Concrete weir and supercritical channel
c. Width 35 feet
d. Length 350 feet
e. Location Spillover left abutment
f. Number and Type of Gates None

OUTLET WORKS: None, except for lake drain

a. Type
b. Location
c. Entrance inverts
d. Exit inverts
e. Emergency draindown facilities 36 inch corrugated metal pipe, 48 inch gate

HYDROMETEOROLOGICAL GAGES: None

a. Type
b. Location
c. Records

MAXIMUM NON-DAMAGING DISCHARGE Unknown

*P.D. indicates plan datum, approximately 183± feet higher than U.S.G.S. datum.

NAME OF DAM: SHERANDO

APPENDIX V

DAM MAINTENANCE INSPECTION REPORT

USDA-FOREST SERVICE

DAM MAINTENANCE INSPECTION REPORT

Ref: FSM 7572.23

1. REGION (3-4)	2. FOREST (5-6)	3. RANGER DIST. (7-8)	4. FOREST INV. NO. (9-12)
08	08	05	0009

5. NAME OF DAM: LOWER SHERANDO LAKE DAM

BLOCK I - MAINTENANCE INSPECTION CHECKLIST

CARD NO. 12

ITEM (Describe deficient items on attached sheets)	NEEDED REPAIRS (By priority) 1	2	None
1. EMBANKMENTS			
a. Slumps, slides			✓
b. Settlement			✓
c. Cracks			✓
d. Seepage			✓
e. Erosion	✓		
f. Slope facing	✓		
g. Debris			✓
h. Traffic damage	✓		
i. Brush, trees			✓
j. Burrows			✓
k. Other			
2. CONCRETE STRUCTURES			
a. Settlement			✓
b. Overturning			✓
c. Heaving			✓
d. Cracks, spalls			✓
e. Joints			✓
f. Undermining			✓
g. Drains			✓
h. Seepage			✓
i. Other			
3. GATES, CONTROLS			
a. Corrosion		✓	
b. Mechanical			✓
c. Structural			✓
d. Clogging			✓
e. Access			✓
f. Other			

ITEM (Describe deficient items on attached sheet)	NEEDED REPAIRS (By priority) 1	2	None
4. CLOSED CONDUITS			
a. Settlement			✓
b. Displacement			✓
c. Cracks, spalls			✓
d. Seepage			✓
e. Clogging			✓
f. Erosion			✓
g. Corrosion			✓
h. Joints			✓
i. Other			
5. SPILLWAYS			
a. Obstructions			✓
b. Erosion			✓
c. Structural			✓
d. Vegetation			✓
e. Other			
6. DOWNSTREAM CONDITION			
a. Backwater			✓
b. Erosion		✓	
c. Bars, pools			✓
d. Boils, piping			✓
e. Other			
7. RESERVOIR			
a. Shore erosion			✓
b. Debris			✓
c. Sediment			✓
d. Other			
8. OTHER *(Identify)*			
a.			
b.			
c.			
d.			

BLOCK II - MAINTENANCE COST ESTIMATE

ITEM OF WORK	UNIT	UNIT COST	QUANTITY PRIORITY 1	QUANTITY PRIORITY 2	COST PRIORITY 1	COST PRIORITY 2
1.(e, f, h) - Heavy erosion tracks both sides of the dam END of working Spillway channel Heavy erosion and debris in downstream channel.					#1000	
3(a) - Gate controls Need sanding and painting						#300
6(b) - Erosion on banks of pool at the end of the spillway - need to stabilize banks with seed, fertilize + mulch.						#500
TOTALS (Enter in Block III, below)					#1000	#800

BLOCK III - SUMMARY MAINTENANCE INSPECTION REPORT

1. DATE OF INSP. (13 18) 04/29/74 MO. DAY YR.	2. HIGHEST PRIORITY CHECKED IN BLOCK 1. (19) 1-e	3. EST. MAINT. COST ($1,000) a. PRIORITY 1 (20 23) 001.0	b. PRIORITY 2 (24 27) 000.8
4. EST. ENGINEER TIME NEEDED (MAN-HR.) a. PRIORITY 1 (28 30) 008	b. PRIORITY 2 (31 33) 008	5. EST. AID & TECH. TIME NEEDED (MAN-HR.) a. PRIORITY 1 (34 36) 080	b. PRIORITY 2 (37 39) 024
6. NOTICE TO OWNER (40) YES - NO	7. DATE OF NOTICE (41 46) --/--/-- MO. DAY YR.	8. LIMITATION (47) YES N NO	
9. TYPE OF LIMITATION (48) L	10. REVISED ESTIMATE OF INSPECTION TIME (MAN-HRS.) a. ENGINEER (49-50) --	b. FOREST OFFICER (51-52) --	c. AID & TECH. (53-54) --

CARD NO. 12

REPORTED BY (Name & signature): Richard Groser | TITLE: Civil Engr | DATE: 4-30-74

V-2

GPO 870-611

APPENDIX VI

CONSTRUCTION AND SOILS REPORT

CONFIDENTIAL AND PRELIMINARY REPORT ON

SOIL INVESTIGATIONS AND CONTROL AT

THE BACK CREEK EARTH DAM

By C. A. Hogentogler, Jr.,
Soils Technician, U. S. Forest Service.

The Back Creek Earth Dam was constructed by the U. S. Forest Service as part of a recreational project located along the north fork of Back Creek, about 15 miles south of Waynesboro, Augusta County, Virginia. In its design and construction were employed the latest principles of soil technology relating to earth dam construction.

Mr. J. C. Dort, Regional Engineer of the U. S. Forest Service, who directed the project, was responsible for the design of the dam and the methods used in its construction. Mr. John W. McNair and Mr. L. R. Strickenberg stationed at Harrisonburg and Mr. C. T. Saunders stationed at the dam site were in immediate charge of the work. Labor was supplied by the Civilian Conservation Corps Camp F-8, under command of Captain Richard Cattlet.

Preliminary investigations of stream flow and foundation conditions were made by W. B. Gallagher of Waynesboro, Va. During

construction the soil investigations were made by an engineer of the Forest Service stationed at the dam site. In this work the U. S. Bureau of Public Roads cooperated by furnishing engineers to assist in the field work and also by making available the testing facilities of their subgrade laboratory at Washington. The engineers cooperating in the field work were Mr. E. A. Willis and Mr. J. A. Kelley, Jr.

Design Features

The first consideration was to provide a spillway so constructed and of sufficient width to prevent overtopping, the estimated cause of approximately 40 percent[1/] of earth dam failures. On the basis of weir readings and flood predictions, a spillway 35 feet wide, excavated into solid rock and separated from the fill by a concrete retaining wall, was considered ample.

An additional requirement was to construct a 36 inch concrete drainpipe at the base of the fill to lower the reservoir during floods. Provision was made to admit water to this pipe by means of a flood-gate controlled from a concrete tower. Both drainpipe and

1/ Earth Dam Projects, by Joel D. Justin, John Wiley and Sons, Inc., New York; Chapman and Hall, London, 1932.

tower are founded on solid rock to eliminate the possibility of settlement. In the design of the dam provision was made to eliminate the three major causes of failure; erosion, due to wave action and possible overtopping, leakages and slides. Rock riprap placed on both slopes and rock wastage which reinforces both toes was to provide protection against erosion. Selected fill material of low permeability, properly compacted, was to insure the structure against detrimental leakage. Slopes with a large margin of safety were to prevent the occurrence of slides.

The embankment as designed, has a maximum width of 225 feet at the base, a maximum height of 30 feet, is 8 feet, 6 inches wide at the crest, and required 25,000 yards of compacted material. There are two berms, one on the upstream side which serves as a foundation for the riprap, and one on the downstream side which serves as a gutter for drainage. The slope on the upstream side is 3:1 from the ground level to the berm and 2½:1 from the berm to the top. The slope on the downstream side is 2:1 throughout. In the finished structure the base is actually much wider than planned, because rock from the spillway excavation has been wasted at both toes.

The fill was constructed by placing material in layers 6 to 8 inches thick and compacting with a sheepsfoot roller. The material for the main portion of the embankment was selected to give a homogeneous structure having low permeability. Wasted material from the spillway excavation of somewhat more porous character was used to a limited extent on the downstream side.

Preliminary soil examinations furnished the data which assisted in the selection of the final location of the dam and suggested certain design features. As part of the construction procedure examinations were made of the character of the natural foundation soil, tests were performed on the borrow pit material and determinations of the moisture content and degree of compaction were made during the placement of the material in the fill.

Investigation of the Foundation

Test pits were used in the examination of the natural soil and star drillings used to explore the underlying rock. A visual examination disclosed that the foundation undersoil was a very loosely compacted sandy loam which was interspersed with gravel deposits, and which, as it approached the rock line, blended into a mixture of sand and gravel. The water table was within two feet of the surface in some places and the rock line was at an average depth of 10 feet.

The drillings which were carried to a depth of 12 feet, indicated about 3 feet of rotten rock underlain by a very hard quartzite. This quartzite was stratified and the drills disclosed the presence of mud seams ranging in thickness from less than 1 inch to a foot or greater. These seams were, for the most part, filled with a very wet, plastic clay, although in pits 5 and 6 gravel or mud was encountered.

Character of the Foundation Soil

The Terzaghi compression tests performed on both undisturbed and disturbed samples disclosed that the permeability of the foundation soil was low enough to indicate that this soil, if disburbed and compacted, would prove satisfactory for use in the embankment. However, the permeability of the soil in undisturbed state was too high for earth dam construction. To have stripped the entire foundation area would have required 13,000 yards excavation and the expense incident to compacting this material in the fill. Therefore, it was decided to prevent the flow of percolating water by excavating a relatively narrow cut-off trench to ledge rock and backfilling with selected material. This trench is 18 feet wide at the bottom, has side slopes of 1:1, and is located 30 feet south of the centerline.

Selection of Material

The first step in the selection of the fill material was to dig test pits at the locations in the reservoir site. From the standpoint of construction this was considered the most desirable location for the borrow-pit. These test pits were made just large enough for a man to work in and were carried to rock, a depth of less than 6 feet in most cases.

Visual examination of these pits furnished pertinent data on the rock line, water table, and depth and character of soil layers. The description of the soil layers used in mapping the soil profile included the color, texture, percentage of gravel (whether rocky, fine etc.), compactness and degree of saturation.

The first step in the testing procedure used in the selection of material for the embankment was to determine the approximate grading of all available material at the reservoir site. By means of these approximate gradings representative samples of material were selected by tests to disclose the properties required in the dam embankment. Samples of those materials which had the desired characteristics were then tested for compaction characteristics required during construction.

Samples of all of the different soil layers were obtained from each test pit and removed to the field laboratory, where they were spread on boards and allowed to dry in the air. The samples were then pulverized and passed through the No. 4 sieve to determine the percentage of rock.

The approximate grading was determined by means of a combined hydrometer and sedimentation analysis. In determining the grading an amount of soil passing the No. 4 sieve required for 50 grams of thoroughly dried soil (corrected for hygroscopic moisture) was first mixed with water and allowed to soak overnight. This mixture was then washed into a 1,000 c.c. cylinder, a deflocculating solution and 15 grams of lead shot were added and the mixture shaken vigorously for five minutes. Water was then added to increase the suspension to a total volume of 1,000 c.cs. after which the mixture was again shaken for five minutes. After periods of 50 sec. and 1 hour respectively, hydrometer readings were made for the purpose of determining the approximate silt and clay contents.

This mixture was then washed through a No. 200 sieve and the approximate grading of that portion retained was determined by means of a simple sedimentation test. The apparatus for this test consists of a glass bottle with a capacity of about a pint which is connected to a burette of 30 c.c. capacity by flexible rubber tubing. The burette is mounted on an electrical buzzer and the bottle is supported by a ring stand.

The material retained on the No. 200 sieve was dried to constant weight, weighed, and transferred to the sedimentation bottle. The system was then filled with water, the bottle was shaken for 1 minute, and the soil allowed to settle in the burette. In order to eliminate the tendency of the soil to stick in the tube the buzzer was used to slightly jar the apparatus while the soil was settling. The volume of the sediment was read at given time intervals.

On the basis of the approximate gradings 8 representative samples were selected for further examination by means of a combined compaction and expansion test.

The compaction test devised by R. R. Proctor 2/ discloses the moisture content at which maximum compaction can be obtained with a sheepsfoot roller during construction. The extent of this compaction is readily ascertained by testing samples at different moisture contents under the impacts of a standard tamper.

The apparatus required for the compaction test, figure 1, consists of a 1/30 cubic foot brass cylinder, 4 inches in diameter and 4 1/2 inches deep, which is mounted on a removable base plate and fitted with a detachable collar 4 inches high to hold the loose soil in place while compacting, a 5 1/2 pound rammer with 3 square inches end area and a plasticity needle which measures the pressure required to force a needle of known end area into the compacted soil. See also figure 1.

2/ Fundamental Principles of Soil Compaction by R. R. Procter, Engineering News-Record, 1933.

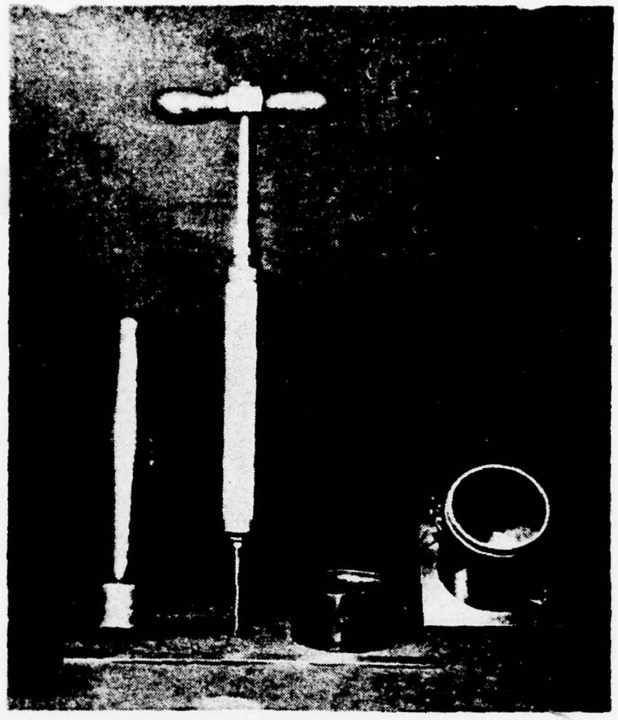


Fig. 1 - Proctor Test Apparatus

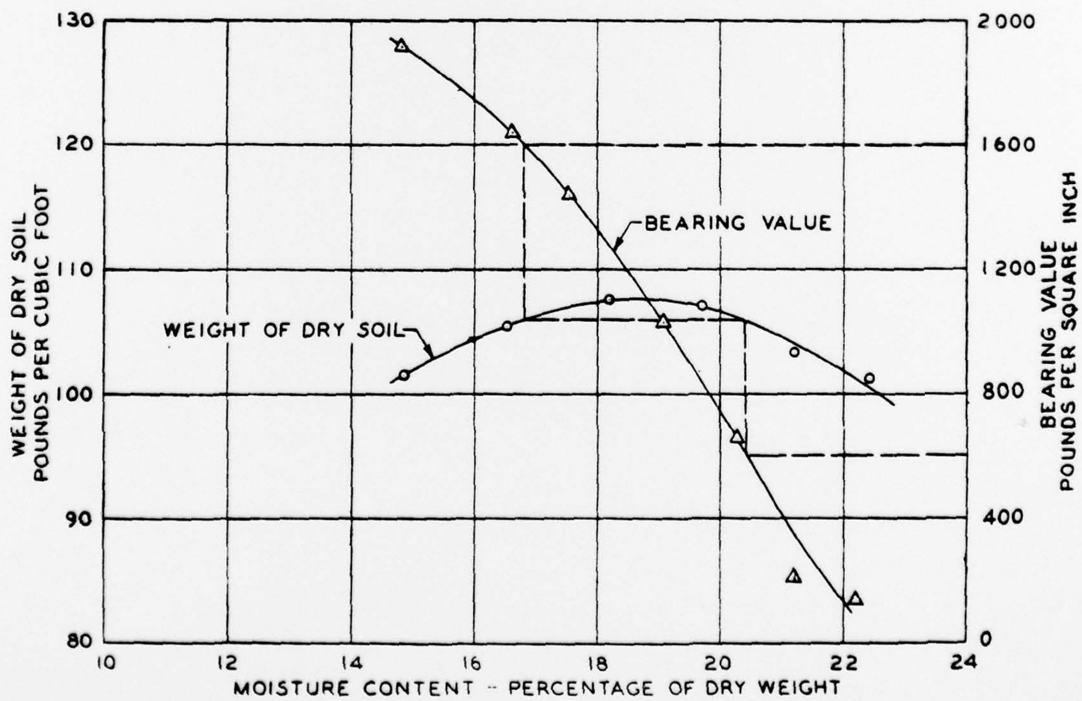


Fig. 2 - Curves obtained from compaction test data

The procedure for performing the compaction test is as follows: Approximately 5 pounds of dry soil passing the No. 4 sieve is thoroughly mixed with just enough water to make it slightly damp and compacted in the cylinder in three layers, each layer receiving 25 blows from the rammer which is dropped from a height of 1 foot above the soil. The soil is then struck off to the level of the cylinder, weighed, and the bearing value determined by the plasticity needle by measuring the pressure needed to force a needle of known end area into the soil at the rate of 1/2 inch per second. A small sample of the compacted soil is oven dried to determine the moisture content.

This procedure is repeated by adding enough water to increase the moisture content about 1 percent each time until the soil becomes very wet and there is a substantial decrease in the wet weight of the compacted soil. The effect of moisture on the compacted densities of the representative samples is shown by plotting the wet and dry densities of the compacted soil, expressed in pounds per cubic foot, against moisture content. The plasticity needle readings, expressed in pounds per square inch, are also plotted against moisture content to show the effect of moisture on bearing value.

One of the curves resulting from the test, shown in figure 2, serves to illustrate the significance of the compaction data. The weight of dry spil-moisture content curve discloses that for this soil a moisture content of about 19 per-cent is required if maximum compaction is to be obtained. The corresponding bearing value is about 1,100 pounds per square inch.

If, at the specified compaction, the bearing value of this particular soil is indicated by the plasticity needle to be higher than 1,100 pounds, the increase can be considered as only temporary if the fill is to be unprotected from water after construction. Thus, a bearing value of 1,600 pounds per square inch indicates a moisture content of slightly less than 17 per cent. This corresponds to a dry weight of about 106 pounds per cubic foot. At this density the soil can take up moisture to a maximum of slightly more than 20 per cent which, in turn, corresponds to a bearing value of but 600 pounds per square inch.

This explains very clearly why fills compacted to a seemingly high degree of firmness may soften and lose stability when subjected to moisture whereas fills compacted to a somewhat less degree of firmness may retain their stability under similar conditions of moisture. It also suggests how uncertain the outcome may be when embankments are constructed with a given

degree of compaction without reference to moisture content or a given moisture content without reference to the type or extent of compaction.

Some conception of the phenomenon of adsorption is necessary in order to understand the full significance of the dry weight-density curves. Such material as presented by Bancroft 3/ suggests that all solids tend to adsorb or condense on their surfaces any gases or vapors with which they are in contact. This adsorption is selective so that for a liquid to wet a solid in the presence of air the liquid must be adsorbed more strongly than the air and must displace the air. Due to adsorbed air on the surface of the solloids drops of rain, after a long period of drought, will often roll along dust without wetting it.

The second characteristic of adsorption to be kept in mind is that the properties of matter in film phase may differ widely from those of matter in the bulk phase. Thus, moisture filling the pores of sandy and silty soils and of clay soils

3/ Applied Colloid Chemistry, by W. D. Bancroft, McGraw Hill Book Company, 1932.

in the plastic state has in general the evaporation and freezing characteristics and the surface tension of water in bulk. As drying or mechanical compaction increases the density of the clay, a moisture content is eventually reached at which the boiling point of the moisture, which now exists as a film, rises, the freezing point lowers and the surface tension increases so that the films become somewhat tougher than water in bulk. This moisture content is termed the plastic limit. Reduction of the moisture content below the plastic limit causes the soil to change from a plastic to a semi-solid material. In thicknesses below two-millionths of an inch the films behave, according to Terzaghi, like semi-solid substances.

The very fine vapor films have an adhesive power so great they cannot be removed from glass by heating at a temperature up to 500° C. nor from soils by forces of 9,000 to 15,000 atmospheres. Due to adsorptive attraction, moisture may enter unconfined soils and separate the particles to such an extent as to cause the soil mass to lose all semblance of stability. In confined soils the entrance of capillary moisture may be productive of enormously high pressures.

When the soil is compacted to maximum density at optimum moisture content the adsorptive attraction between water and soil particles is probably completely satisfied so that the tendency for moisture to enter the soil and expand or soften the soil mass is largely eliminated. However, if during the process of compaction the soil grains have been placed in a condition of elastic stress they can be expected to rebound and cause the soil mass to expand. Such tendency to expand as well as the compressible characteristics of compacted samples under conditions of changing loads or moisture content may be determined by testing samples of compacted soil in the Terzaghi compression test apparatus.

The relative resistance of wet and dry soil samples to the entrance of capillary moisture is demonstrated by experiments reported in PUBLIC ROADS, June 1931.

Two disks cut from each of a number of compressed soil samples were immersed in water -- one in the wet state and the other after being dried to constant weight. Sixty-seven of the disks immersed in the wet state remained intact after being immersed for an average period of nine months. The corresponding disks, immersed in the dry state, disintegrated after being immersed for periods ranging from 10 minutes to 1 hour.

The compacted materials were found to have low permeability and but little tendency to expand appreciably under conditions to be met in the fill. The average permeability of the compacted material was such as to indicate a flow of about 0.005 gallons per day per square foot of case wall face.

Results of the routine subgrade tests performed on the representative samples are shown in table I.

TABLE I.

Sample No.	Mechanical Analysis: Particles smaller than (mm) 2.0	0.42	0.05	0.005	0.001	Constants of fraction smaller than 0.42 mm: Liquid Limit	Plasticity Index	Shrinkage Limit	Shrinkage Ratio	Field Moisture Equivalent
1-B	80	73	46	28	14	35	14	23	1.7	31
2-A	76	71	46	21	11	30	12	19	1.7	24
8-A	60	50	30	17	8	33	12	24	1.7	28
16-A	86	84	60	37	20	38	17	23	1.7	28
17-B	78	70	45	27	14	35	16	19	1.7	25
20-B	71	67	53	40	29	55	26	25	1.6	41
21-A	79	73	55	33	17	37	17	20	1.8	28
26-B	100	100	58	38	19	37	17	20	1.6	29
28-A	100	100	58	33	16	32	12	23	1.7	25

According to table I all of the soils may be considered to have enough sand to produce stability, enough clay to bind the sand in a water-tight mass, and not enough silt to cause detrimental capillary rise. The low liquid limits for soils of this grading indicate the absence of mica, diatoms or organic matter. The plasticity indexes are high enough to indicate that

the soils will not lose stability in the presence of water without manipulation. The shrinkage limits are well below the liquid limits, a condition usually met in the more stable soils. The field moisture equivalents are also well below the liquid limits, a further evidence of stability.

It is interesting to note that the optimum moisture contents of all of the soils selected for the fill were just slightly above their plastic limits. This is highly significant from the standpoint of practical construction and suggests a definite basis for the selection of dam soils. To begin with, we are concerned with the mixing water both as a lubricant and as an adhesive. Its lubricating properties control the effort required to produce a given degree of consolidation. Its adhesive properties determine the stability of the compacted soil. When the surface tension of the moisture films increases after the moisture content is reduced below the plastic limit the lubricating properties of the films are reduced and the adhesive properties increased. As a result the plastic limit is the moisture content of the soil below which there is a considerable increase in the compaction required to produce a given density and also below which the stability of the soil is considerably increased.

Therefore, it seems desirable from a practical standpoint to compact the soil in the plastic state but as near to the semi-solid state as possible. Consequently it would seem

that for optimum moisture contents just above the plastic limit there is obtained the most favorable balance between lubricating and adhesive properties.

If the optimum moisture content of a soil is just slightly above the plastic limit the evaporation during consolidation will probably cause the soil to be just at or slightly below the plastic limit at the conclusion of rolling. It appears at present that the most suitable soils have optimum moisture contents just slightly above the plastic limit and that the difference between the plastic limit and the optimum moisture content serve as a basis for the selection of fill soils, although more work must be done along this line before any specifications could be set up.

Control During Construction

During construction it was necessary to perform a number of tests in the field to control the moisture contents and the degree of compaction. The moisture contents were quickly determined by packing the wet soil in the Proctor cylinder, weighing, measuring the bearing value and comparing these values to the wet weight and bearing value curves for the particular sample. These values were repeatedly checked by oven drying small samples to

determine the moisture contents more accurately. If the soils were found to be below the optimum moisture content they were sprinkled with water on the fill; if found to be above the optimum they were spread on the fill in thinner layers and allowed to dry either before rolling or during longer rolling periods.

The extent of compaction was readily determined by a simple field test which measured the compacted density of the soil layer in place. This test was performed in the following manner: Approximately 15 pounds of the compacted soil was removed from the layer by a post hole auger, and weighed on a milk scale of 30 pound capacity. The same soil was then packed in the Proctor cylinder, weighed and the density and moisture content computed as in the regular tests. The hole was filled with sand of a known loose weight per cubic foot and the volume computed. The wet and dry densities of the compacted material were determined from the weight, moisture content and volume. A comparison of the densities of the wet soil in place and the same soil compacted in the cylinder in the standard manner immediately indicated whether more rolling was required or whether the number of trips could be decreased.

The extent of compaction was determined more quickly by measuring the bearing value of the soil layer in place, and comparing the values to the readings obtained on the same soil compacted in the standard manner in the cylinder. If the bearing values of the soil layer were less than those for the soil in the cylinder more rolling was required. This test was not used as extensively as the dry weight test because of the presence of gravel in the material which interfered with the needle.

Construction Procedure

During construction the stream flow was taken care of by a 36 inch metal pipe encased by a 6 inch thickness of concrete supported on ledge rock.

Water was controlled by a flood gate which was mounted on a tower at the upstream end of the pipe. This tower was also founded on rock.

To prevent the creeping of water along the drainpipe several collars were added and very carefully selected material was hand-tamped around the pipe and the collars.

Water was carried to the drainpipe by a trench, which intercepted the stream about 500 feet in front of the dam site. The stream was diverted into this trench by a cofferdam which consisted of truckloads of material dumped across the stream bed.

After the stream flow had been diverted through the drainpipe and the dam site dried up considerably the excavation of the cut-off trench was begun. The east side of the trench was dug by hand and the dirt removed by a scraper and reworked on the back of the fill. The rock foundation for the core wall was carefully cleaned and dried, in some cases, by compressed air before any material was placed.

The west side was excavated with a shovel and the rock cleaned off by hand. When this was finished a number of mud seams were noticed and some of the rock was removed in an attempt to reach a more solid foundation. As no reduction in the number or size of the mud seams was noticed after two feet of rock had been removed the excavation was discontinued but, in order to eliminate the possible washing of the core wall material in contact with these seams, they were capped with concrete.

After the trench had been completely cleaned and dried the placement of material was begun. The borrow pit area was first cleared of all stumps and the top soil carefully stripped

off. The soil was tested for moisture content by means of the plasticity needle by the procedure outlined above and then loaded into trucks by the gas shovel and hauled to the dam. The efficiency of the shovel was greatly increased by the assistance of the trail-builder in pushing the soil into the location of the shovel and so deepening the cut.

The material was spread with a bulldozer in 8 inch layers. The soil was compacted by means of two sheepsfoot rollers drawn by tractors. One of the tractors was equipped with a bulldozer and was used to spread the layers. Generally 8 to 10 trips of the roller were required to produce the desired compaction.

The effectiveness of this method of consolidation for furnishing the soil with a high resistance to erosion was clearly demonstrated during a flood which occurred when the embankment was but partially completed. During this period water flowed over the dam to a maximum depth of possibly three feet for a period of several hours without causing serious erosion. That the adsorptive attraction of the compacted soil was completely satisfied is indicated by the fact that moisture determinations made on the compacted soil after prolonged soaking failed to disclose any increase over that at which the embankment was constructed.